

# Design Basis for Offshore Wind Structures

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## Summary:

For development of offshore wind farms, the design basis will serve as a valuable tool. The purpose with the design basis is twofold – it serves as a technical basis for design and construction of the wind farm and the design basis may together with the contract serve as a basis for managing project variation orders.

Experience from the offshore oil & gas industry have shown that a good design basis will allow for a fast track development project of high quality with only minor variation orders, while a poorly worked out design basis opens up for delays and major variation orders.

The present paper describes good practice for a project design basis based on DNV involvement in more than 25 offshore wind farm projects. Among other things the design basis should cover following technical aspects:

- Site specific data
- Interface between wind turbine and support structure
- Design standards
- Design analyses and conditions
- Manufacturing
- Installation
- Commissioning
- Operation & Maintenance

The minimum requirements for some of the above aspects will be described in detail in the paper. Special focus will be on requirements to the integrated load model, where the flexibility of the support structure shall be taken into account in the load calculation of the wind turbine. Based on recent experience other problem areas like splash zone coating and wave slamming loads will be highlighted and guidance will be given.

**Key words:** offshore support structures, design basis.

## 1. Introduction

As investments in offshore wind farms become larger DNV believes that there will be an increased focus on reliability and safety from all stakeholders. For offshore wind turbines and support structures, DNV anticipates an even greater focus in these areas due to much higher maintenance and replacement costs when compared to onshore applications. DNV has therefore developed *Design of Offshore Wind Turbine Structures, DNV-OS-J101* [1] and *Design and Manufacturing of Wind Turbine Blades* [10].

The intention with the present paper is to give experience feed back on selected topics related to the design basis for the offshore support structure, knowing that most of the presented topics are not new to the offshore industry, but may be new in an offshore wind context.

The topics covered in this paper are:

- Determination of design wave
- Specification of turbulence intensity for design
- Combination of wind speed and wave sea states
- Integrated load model
- Ice loads
- Corrosion and coating schemes
- Qualification of grout material
- Slamming loads from waves

The requirements to the geotechnical analysis, focusing on the site assessment of the soil parameters are described in a separate paper [11].

## 2. Determination of Design Wave

For determination of design waves it is necessary to have measured reliable data for a sufficient period of time both in calendar time and with regard to coverage. The design wave height should be determined based on a statistical analysis of measured site-specific wave data or data obtained by hindcast studies.

The wave climate is represented by the significant height  $H_s$  and the spectral peak period  $T_p$ . In the short term, i.e. over a 3-hour period, stationary wave conditions with constant significant wave height and peak period are assumed to prevail. Wave statistics are to be used as a basis for representation of the long-term and short-term wave conditions. Empirical statistical data used as a basis for design must cover a sufficiently long period of time.

Wave data obtained on site are to be preferred over data observed at an adjacent location. Measured wave data are to be preferred over visually observed wave data. Continuous records of data are to be preferred over records with gaps. Longer periods of observation are to be preferred over shorter periods. When no site-specific data are available and data from adjacent locations are to be capitalised on instead, proper transformation of such other data shall be performed to account for possible differences due to different water depths and different seabed topographies. Such transformation shall take effects of shoaling and refraction into account. Hindcast of wave data may be used to extend measured time series, or to interpolate to places where measured data have not been collected. If hindcast is used, the hindcast model shall be calibrated against measured data to ensure that the hindcast results comply with available measured data.

The method for determination of the extreme design wave is given in the following. The extreme design wave height is the wave height with recurrence period  $T_R=50$  years,  $H_{50}$ . The significant wave height  $H_s$  is stationary in  $T_S=3$  hours and the arbitrary  $H_s$  is typically Weibull-distributed

$$F_{H_s}(h)=1-\exp(-(h/h_0)^k)$$

The annual maximum of  $H_s$  may in the upper tail be approximated by a Gumbel distribution:

$$F_{H_s,\max}(h)=F_{H_s}(h)^N \approx \exp(-\exp(a(h-b)))$$

where  $N=2920$  is number of 3-hour sea states in one year.

The 50-year significant wave height is the 98% quantile in the distribution of the annual maximum

$$H_{S50} = F_{H_{Smax}}^{-1}(1-1/50) = F_{H_{Smax}}^{-1}(0.98) = b - \ln(-\ln(0.98))/a$$

An approximation can be obtained as the  $(1-T_S/T_R)$  quantile in the distribution of the arbitrary  $H_S$

$$H_{S50} \approx F_{H_S}^{-1}(1-T_S/T_R) = h_0(-\ln(T_S/T_R))^{1/k}$$

By Stokes 5th order theory, the 50-year wave height for design becomes

$$H_{50} = 1.86 H_{S50}$$

which is well known from the southern part of the North Sea. The values obtained by the above method should be considered as central estimates when the underlying distribution function is determined from limited data and is encumbered with statistical uncertainty.

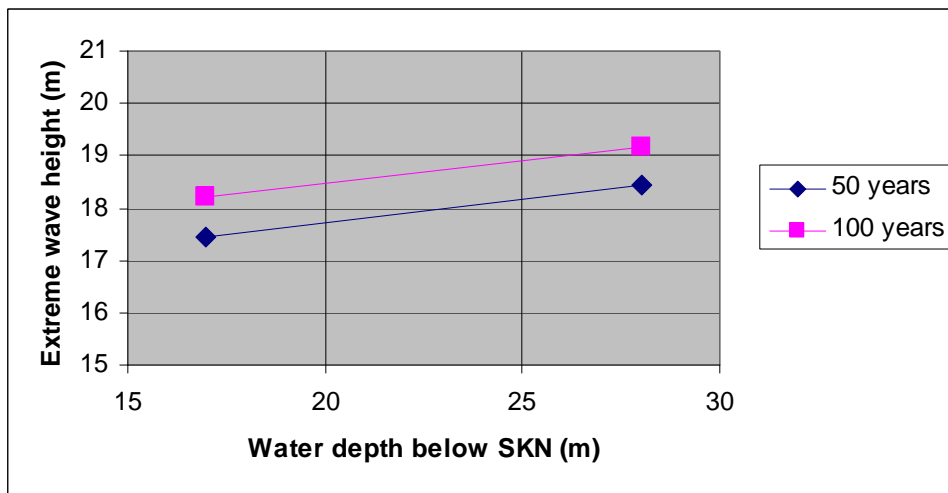


Figure 1 - Example: Central estimates of 50- and 100-year wave heights vs. depth below MSL for a location in the German Bight.

### 3. Turbulence Intensity

The long-term distribution of the 10-minute mean wind speed and the short-term turbulence data conditional on the 10-minute mean wind speed should be calculated by use of measured site specific data. If no measured site specific data is available, the same assumptions as for the wave data apply.

An example of the empirical distribution of the turbulence intensity  $I_T$ , defined as the ratio  $\sigma_U/U_{10}$ , where  $\sigma_U$  denotes the standard deviation of the wind speed and  $U_{10}$  denotes the 10-minute mean wind speed, is shown in figure 2 in terms of the median, the mean value and the 90% quantile of the turbulence intensity as functions of  $U_{10}$ .

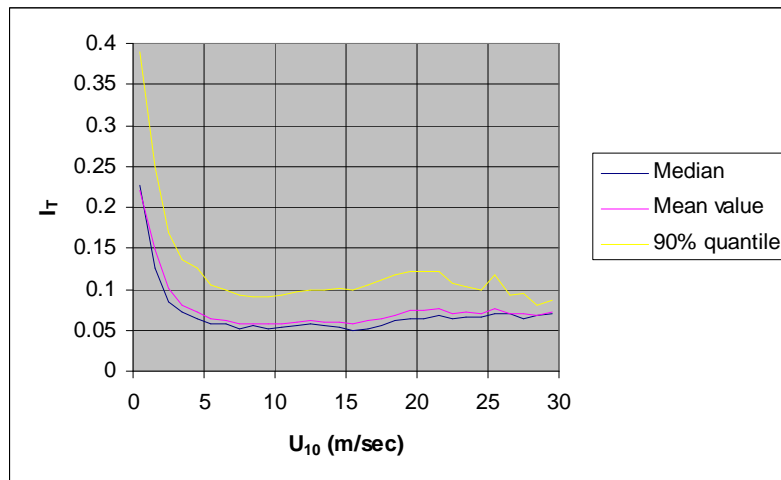


Figure 2 - Example. Turbulence intensity (background)

For design purposes, in the evaluation of fatigue loads, the turbulence intensities given at each wind speed must be the 90 % quantile of the turbulence intensity distribution. The influence of wake turbulence can become critical and should therefore be considered in addition

#### 4. Correlation between Wind Speed and Sea State

For an integrated load analysis it is required to have information regarding the correlation between hub height wind speed and sea state. This correlation is according to [1] given as the most probable sea state occurring at a given wind speed.

A pragmatic approach to finding the correlation between wind speed and wave height will typically have to be chosen since no direct measurements are available. The chosen approach should however be documented and validated with measured data, if available.

Furthermore it is important to have information regarding the wind and wave misalignment. Previous experience shows that in most cases it will be conservative to apply the wind and wave loads aligned at all time, but in order to use this assumption it is necessary to have information about the duration and magnitude of wind and wave misalignment.

#### 5. Integrated Load Model

For a detailed description of the integrated load model and analysis taking into account the flexibility of the foundation in the ULS and FLS load analyses reference is made to e.g. [2].

In ref. [2] semi-integrated approaches are described for large wind turbines, i.e. for offshore turbines where the wind load is dominating. The basis is application of an equivalent monopile foundation substituting the real jacket type foundation structure including tower, support structure and soil. The properties for the equivalent monopile are equal to the real structure with regard to stiffness, mass and hydrodynamic behaviour. Approaches for superposition of loads are described and the results are presented and discussed.

Engineering software packages for fully integrated load analysis are not yet commercially available and semi-integrated load analyses are typically based on in-house developments on programs from the offshore and the wind industry. This development is expected to continue as offshore projects

are planned further offshore, turbines are getting larger and support structures are getting more complex.

### 6. Ice Loads

For sites located in areas with presence of ice like e.g. the Baltic Sea, ice loads will often be a design driver. Ice loads may be calculated in accordance with e.g. [1] or [4].

Loads from laterally moving ice shall be based on relevant full scale measurements, on model experiments which can be reliable scaled, or on recognised theoretical methods. When determining the magnitude and direction of ice loads, consideration is to be given to the nature of the ice, the mechanical properties of the ice, the ice-structure contact area, the size and shape of the structure, and the direction of the ice movements. The oscillating nature of the ice loads, including build-up and fracture of moving ice, is to be considered.

Horizontal loads from moving ice should be considered to act in the same direction as the concurrent wind loads. The water level to be used in conjunction with calculation of ice loads shall be taken as the high water level or the low water level with the required return period, whichever is the most favourable.

Ice loads on inclined structural parts such as ice-load reducing cones in the splash zone may be determined according to Ralston's formulas, as described in [1].

Relevant statistical data for the following sea ice conditions and properties shall be considered:

- geometry and nature of ice
- concentration and distribution of ice
- type of ice
- mechanical properties (compressive strength and bending strength)
- velocity and direction of drifting ice
- thickness of ice

Observations of ice may be found in ice atlas, handbooks etc, however more investigations in potential offshore wind farm areas could give more confidence in determination of ice loads.

### 7. Corrosion and coating schemes

Due to recent experience it has become clear that more attention should be given to corrosion protection of offshore wind structures.

The following 3 methods are commonly used for protection against corrosion:

- Corrosion allowance (additional wall thickness)
- Cathodic protection (anodes, impressed current)
- Coating

#### 7.1 Recent offshore experience

##### 7.1.1 Coating

The coating schemes listed below are typically applied for offshore oil & gas platforms in the North Sea, ref. NORSOK M-501. It should be emphasized that qualification of coating products,

applicators and coating application procedures as well as inspections during coating application and in-service life is an integral part of the system.

### *Splash zone*

1.5 mm multi-layer glass flake epoxy or polyester, alternatively 450 micron multi-layer high solid epoxy or 200 micron thermally sprayed aluminium (TSA) with a sealer coat. To our knowledge no standards have yet been published covering glass flake systems, which usually are applied according to manufacturer's recommendations. TSA coating is specified in DIN8566/2, ISO2063 and BS2569.

The splash zone is for the North Sea oil & gas fields typically defined as LAT -4 m to +6 m depending on sea environment. For coastal regions the splash zone could be even larger.

### *Above splash zone*

- Zinc based primer
- Tie-coat
- Intercoat epoxy based
- Topcoat Polyurethane

Total dry film thickness 275 – 335 micron depending on coating system applied.

### *Submerged zone*

450 micron multi-layer high-build epoxy (mastic type) together with cathodic protection. Alternatively the corrosion protection system consists of cathodic protection only, and in that case no coating system is applied in the submerged zone.

The above systems have been used for a number of years together with a systematic approach for supervision during application, requirements to application, inspection after application, reporting, certification of final coated structure. The structures are in addition inspected in-service annually, repairs are made if necessary and the entire process is part of the certification by an independent 3<sup>rd</sup> party. Detailed requirements to pre-qualification, application etc. may be found in e.g. NORSOK M-501.

Present experience has shown that application of coating needs focus. Damage to the coating is typically a result of poor application or poor conditions for application and will lead to high maintenance costs. This could be avoided by inspection by owner's representative during manufacturing.

Presently, offshore wind turbine structures do not normally follow the above coating and maintenance schemes. For some offshore wind turbine monopile structures coating damages have been reported for the splash zone area after only a few years time and corrosion has started to develop. More focus on splash zone coating is therefore needed.

### *7.1.2 Corrosion Allowance*

Corrosion allowance is typically used as protection in addition to the coating system. For a jacket structure in service with the offshore oil & gas industry 12 mm is normally added to the wall thickness in the splash zone in order to cover the planned 25 years operating life. For offshore wind structures 6 mm are normally added to cover the planned 20 years operating life, ref. DNV-OS-J101. The reduced corrosion allowance required at present for offshore wind structures is justified by the difference in structural safety class in conjunction with observations made for 30 years old oil & gas North Sea platforms.

### 7.1.3 Cathodic Protection

Cathodic protection systems where sacrificial anodes are corroded away over the life time instead of the structural steel, have worked well on offshore oil & gas platforms for more than 25 years.

## 7.2 Structural design considerations

### 7.2.1 FLS versus ULS driven design

To understand the requirements to a corrosion protection system some structural design considerations are described below. The considerations are based on a 4-legged jacket and a monopile foundation structure, both steel structures.

Jacket legs and braces are typically designed by Ultimate Limit State (ULS) in the splash zone in contradiction to for instance offshore wind turbine monopile foundations, where the design mainly is determined by the Fatigue Limit State (FLS). Jacket legs and braces are typically utilised 90 to 100 % in ULS and have fatigue lives well in excess of the design requirement. Tubular joints in these structures are typically not located in the splash zone. Offshore wind turbine monopile structures typically have a fatigue life of 20 to 40 years at girth welds and attachment welds in the splash zone, i.e. close to the design requirement of 20 years.

The wall thickness is for fatigue calculations normally reduced by half the corrosion allowance in the splash zone – the beneficial effect of this increase in cross section is however rather limited compared to the negative effects of corrosion.

Free corrosion will for ULS governed structures merely lead to a reduction of the active area in the stress calculations, and for jacket legs and monopiles therefore have only a limited influence. For FLS governed structures there will be the same reduction of the active area, but more importantly there will for free corrosion be a shift in S-N curve leading to significantly shorter fatigue lives.

Structural details that may become critical when subject to a shift in S-N curve are typically heat affected zones like:

- Tubular girth welds
- Tubular longitudinal welds
- Monopile attachment welds for J-tubes, boat landings, ladders etc.

Uniform corrosion of the structure will have the following consequences:

- reduced cross section for stiffness and stress calculations
- shift in S-N curve (does not cover pitting corrosion)

The shift in S-N curve will reduce the fatigue life by a factor of around 4.

Corrosion reduces the fatigue life as the crack initiation period, which corresponds to approximately 30-40 % of the fatigue life, is eliminated by the corrosion process together with a more rapidly developing crack growth phase. When repaired it is therefore essential that all corrosion is removed and that the structure is intact after the repair.

## 7.3 Offshore wind application

It can from a structural point of view be concluded for offshore wind farms that:

- The design life for the coating system should as far as possible correspond to the design life of the wind farm and with minimum maintenance requirements.

- The coating application should be monitored carefully
- Coating inspection system should be set up (annual inspections)
- Coating should be repaired if necessary

It is recommended that the performance of the applied coating system is documented and proven, e.g. by prequalification in accordance with Norsok standard M-501 or by operational experience. Further, it is recommended to follow up during application with regard to surface preparation, air humidity, temperature, layer thickness etc. following rules and guidelines in e.g. DNV-OS-J101, DNV-OS-C401, Norsok standard M-501 or ISO12499-7.

Based on a review of existing coating systems DNV can therefore recommend the following for offshore wind farms:

Atmospheric zone:	Coating system in accordance with ISO 12944-5, C5-M, Systems S7.09 or S7.14
Splash zone:	Glass flake reinforced epoxy (min. 1.5 mm DFT), Glass flake reinforced polyester (min. 1.5 mm DFT) or thermally sprayed aluminium with silicone sealer (min. 200 micron DFT)
Submerged zone:	Multilayer two component high build epoxy (min. 450 micron DFT). Alternatively cathodic protection only.

For application of the coating there are a number of requirements with regard to:

### Surface preparation

- Coating application
- Qualification requirements
  - Pre-qualification of products
  - Qualification of companies and personnel
  - Qualification of procedures
- Inspection and testing
- In-service inspection

Detailed requirements are described in e.g. NORSOK M-501.

## 8. Qualification of Grout Material

The grout materials for grouted connections shall comply with relevant requirements given for both concrete and grout in ref. [7] as well as [1].

The grouting materials shall have sufficient workability to secure filling of the annulus without establishing air pockets or water pockets or both in the grout.



Test specimens are to be made with varying mix proportions to simulate the batching tolerances under field conditions. Grout mixes shall as a minimum be tested for the following properties:

- Density
- Air content
- Workability
- Viscosity
- Stability (separation and bleeding)
- Setting time
- Compressive strength.
- Shrinkage/expansion
- Effect of admixtures and compatibility of admixtures.

If no sufficient documentation of the material properties of the grout is available, experimental verification of the properties must be carried out. A sufficient number of tests must be made in order to perform a statistical evaluation of the results and establish design values. Grout manufacturers must expect such documentation of design parameters to be presented in a transparent way in new markets like e.g. Germany where high strength grout is not currently approved as a construction material and there will be a review process prior to approval.

Samples for testing of the grout quality shall preferably be taken from the emerging, surplus grout. If this is not possible, other means of monitoring the density of the return grout are to be provided.

Test on grout samples shall be carried out in order to verify the characteristic compressive strength of the grout. The characteristic compressive strength is normally defined as the compressive strength after setting 28 days at 20 degrees C. If the grout is to be subjected to loading before the characteristic design strength has been achieved, for example due to installation of other structures or due to wave and wind loading before 28 days have passed, the assumed allowable grout strength at the time of the loading shall be verified. Curing of the specimens shall take place under conditions which are as similar to the curing conditions of the placed grout as possible.

The compressive strength is normally to be tested by making sets of 5 test specimens each. One such set of 5 specimens shall be used every time a test is to be carried out. Each specimen is to be taken from a single, random sample. The total number of test sets required according to this specification shall be obtained for every consumed 200 m<sup>3</sup> of grout, once per shift or once per grouted compartment/annulus, whichever gives the largest number of test specimens. The test specimens are to be adequately marked and recorded for identification.

Acceptance criteria for the grout properties will depend on the actual design of the grouted connection.

### **9. Slamming Loads from Waves**

Horizontal members in the splash zone are susceptible to forces caused by wave slamming when the member is being submerged. The dynamic response of the member should be accounted for.

For a horizontal member the slamming force per unit length may be calculated as

$$F_s = 1/2 \rho C_s D v^2$$

where

$F_s$  = slamming force per unit length in the direction of the velocity

$\rho$  = mass density of sea water

$C_s$  = slamming coefficient

$D$  = member diameter

$v$  = velocity of water surface normal to the surface of the member.

For smooth circular cylinders the slamming coefficient should not be taken less than 3.0.

Since the slamming force is impulsive, dynamic amplification must be considered when calculating the response. For a horizontal member fixed at both ends, dynamic amplification factors of 1.5 and 2.0 are recommended for the end moments and the midspan moment, respectively.

The fatigue damage due to wave slamming should be considered as well and may be determined by the procedure given in e.g. [6]. The calculated contribution to fatigue due to slamming has to be added to the fatigue contribution from other variable loads.

All bolted connections should be pre-tensioned as slip bolts will not have sufficient fatigue life.

## 10. Conclusion

The paper has presented selected topics related to the design basis for offshore support structures, knowing that most of the presented topics are not new to the offshore industry, but may be new in an offshore wind context. The intention has been to give *experience feedback* based on real life experience with more than 25 offshore wind projects in order to improve future projects.

The paper has pointed at some areas where more work has to be done in order to develop the planned offshore wind farms. Such areas are:

- Methods for combination of wind speed and wave sea state
- Development of a commercially available software tool for fully integrated load analysis
- Observation of ice and testing for determination of ice loads
- Qualification of high strength grout material through relevant building authority (in e.g. Germany)

For other areas like determination of design wave, coating schemes, wave slamming loads etc. it is more a question about utilising existing experience within the offshore industry. For such areas the DNV set of standards provides detailed guidance transferring and consolidating the vast amount of international offshore experience to the offshore wind industry.

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